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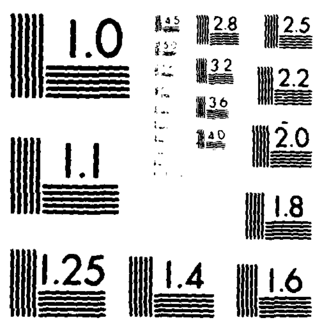

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DETERMINING PEAK-DISCHARGE FREQUENCIES IN AN URBANIZING WATERSHED: A CASE STUDY

by

STEVEN F. DALY
JOHN PETERS

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A case study is presented of a hydrologic investigation of the Red Run Drain-Lower Clinton River watershed, an area near Detroit, Michigan, that has undergone urbanization since the 1940's. The purpose of the study was to determine peak-discharge frequencies at gaged and ungaged locations for existing and future conditions. Population density was used as an indicator of urbanization in relationships defining unit hydrograph parameters and hydrologically significant impervious area. Input parameters for a (Continued)

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single event rainfall-runoff simulation model (HEC-1) were developed to reflect watershed conditions in the years 1940, 1950, 1960 and 1975. The input parameters were verified by reconstructing observed flood events that occurred at these points in time. Sets of synthetic winter and summer storm hyetographs were input to HEC-1 to develop a series of curves for two gaging stations that relate peak discharge to magnitude of synthetic storm for each watershed condition. The curves were used to transform the series of recorded annual peak discharges at each gage to a stationary series that reflects 1975 watershed conditions. Discharge frequency estimates were then developed for ungaged locations using winter and summer synthetic storms that were assigned exceedance frequencies consistent with actual exceedance frequencies at the gaged locations. Projections of future population density were the basis for developing HEC-1 input parameters representing year 2000 and 2025 watershed conditions. Estimates of peak discharge-frequencies for the future conditions were made at the gaged and ungaged locations using the methods described above.

DETERMINING PEAK-DISCHARGE FREQUENCIES IN AN URBANIZING WATERSHED - A CASE STUDY

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Abstract. A case study is presented of a hydrologic investigation of the Red Run Drain-Lower Clinton River watershed, an area near Detroit, Michigan, that has undergone urbanization since the 1940's. The purpose of the study was to determine peak-discharge frequencies at gaged and ungaged locations for existing and future conditions. Population density was used as an indicator of urbanization in relationships defining unit hydrograph parameters and hydrologically significant impervious area. Input parameters for a single event rainfall-runoff simulation model (HEC-1) were developed to reflect watershed conditions in the years 1940, 1950, 1960 and 1975. The input parameters were verified by reconstructing observed flood events that occurred at these points in time. Sets of synthetic winter and summer storm hyetographs were input to HEC-1 to develop a series of curves for two gaging stations that relate peak discharge to magnitude of synthetic storm for each watershed condition. The curves were used to transform the series of recorded annual peak discharges at each gage to a stationary series that reflects 1975 watershed conditions. Discharge frequency estimates were then developed for ungaged locations using winter and summer synthetic storms that were assigned exceedance frequencies consistent with actual exceedance frequencies at the gaged locations. Projections of future population density were the basis for developing HEC-1 input parameters representing year 2000 and 2025 watershed conditions. Estimates of peak discharge-frequencies for the future conditions were made at the gaged and ungaged locations using the methods described above.

Introduction

Determining the impact of urbanization on flood flow frequency is an area of substantial interest for flood control planning and design and for studies required by the national flood insurance program. This paper reports methodology used in a Corps of Engineers study (7) to evaluate channel modification proposals for the Red Run Drain, which is located immediately north of Detroit in the Clinton River basin. The study was conducted to determine peak-discharge frequency curves at two gaged and numerous ungaged locations for existing (1975) land use conditions as well as for conditions in the years 2000 and 2025. Methodology is reported herein for converting a nonstationary time series of annual peak discharges to a stationary time series representing a specific set of land use conditions, and procedures are described for developing peak-discharge frequency curves at ungaged sites.

Previous Studies

A number of studies relating to the hydrology of Southeastern Michigan have been reported (1, 2, 3, 4, 5, 6). In particular, Brater and Sherrill (2) provided the results of an extensive project initiated in 1964 to develop a procedure for estimating frequency of storm runoff for small basins in various stages of urbanization. Results of that project were used in a number of instances in the study described in this paper.

*Paper presented at the International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, July 23-26 1979.

Watershed Characteristics

The Clinton River Watershed covers an area of 760 square miles in the southeastern portion of the lower peninsula of Michigan. (Figure 1) The Clinton River Watershed includes both rural and urban areas, and roughly resembles a maple leaf, with the stem or base the outlet to Lake St. Clair. The Main Branch of the Clinton River originates in the hilly country of Oakland County, where a great number of lakes, ranging in size from a few acres to several square miles, remain as a legacy of the last period of glaciation. It is from these lakes that the river obtains much of its base flow. The Main Branch flows in a general southeasterly direction for 69 miles, through several smaller urban areas (Pontiac, Rochester, Utica), where it is joined by the North and Middle Branches, which drain relatively rural areas of 201 and 77 square miles, respectively. The confluence of the three tributaries, known locally as the "Forks Area," produces the Clinton River proper, which meanders through a level plain in a generally easterly direction for 11 miles, through the City of Mt. Clemens, to its outlet in Lake St. Clair.

Four miles upstream of the Forks Area the Main Branch is joined by the Red Run Drain. The Red Run, which drains an area of approximately 140 square miles, originates in the heavily urbanized northern suburbs of Detroit, and flows in a northeasterly direction through the rapidly urbanizing sections of western Macomb County. (Figure 2)

Three distinct types of topography are evident in the Clinton River Watershed: The hilly, lake covered region that comprises the head waters of the Main Branch; the rolling, well-drained country that comprises the watershed of the Middle and North Branches; and the flat, poorly drained areas that extend in a broad plain along Lake St. Clair and comprise the watershed of the Red Run. It is in this area of poorly drained glacial till that the majority of development in the watershed has taken place. The Red Run Drain and it tributaries have been ex-

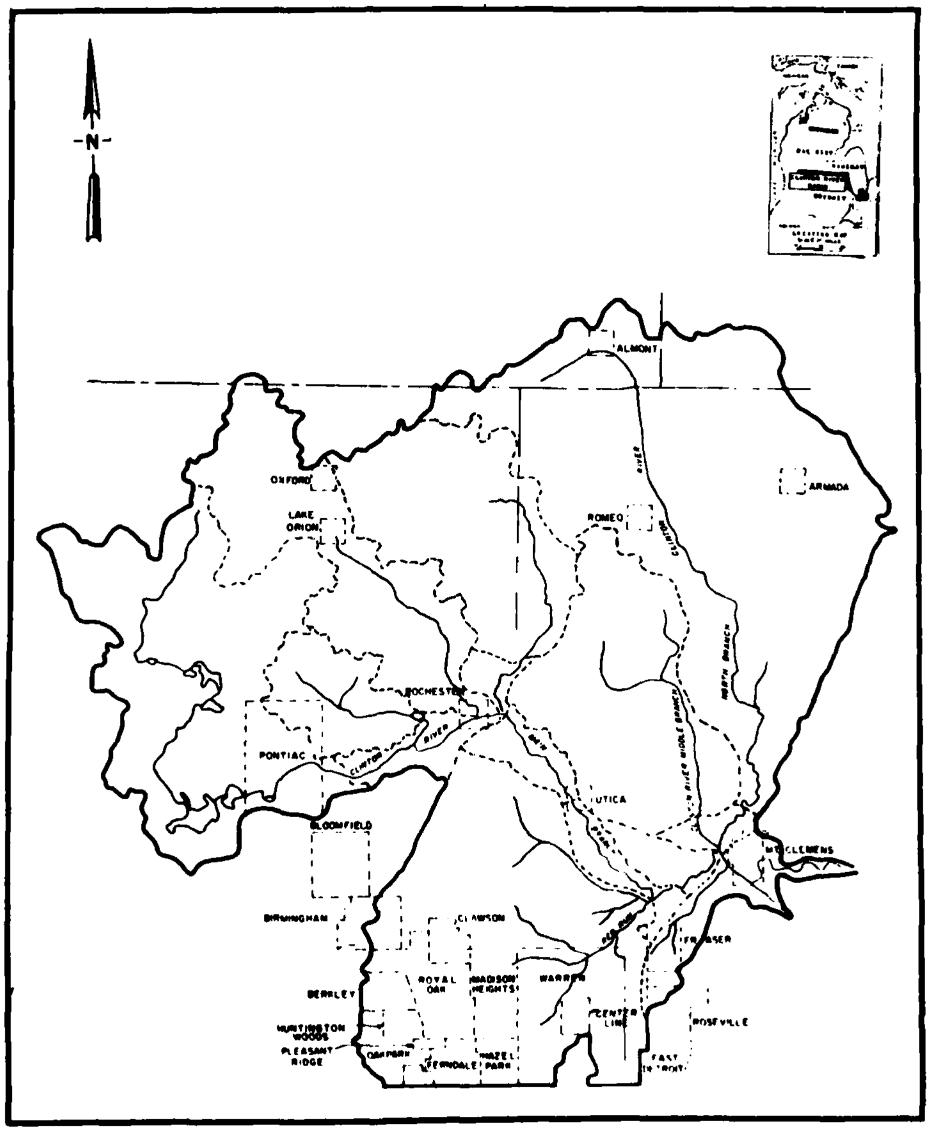

Figure 1. Clinton River Watershed

tensively modified to provide adequate capacity for storm drainage from the urban areas. However, the rate of urbanization has exceeded the rate of drainage improvement in recent years and wide-spread overland and basement flooding has resulted.

Summary of Available Data

Meteorologic and streamflow records for the Clinton River Basin are relatively comprehensive and provide excellent coverage of the more recent floods. Precipitation records for the City of Mt. Clemens, considered typical of the basin, are available from about 1897 to the present. Hourly rainfall data are available for the period 1960-1975 from the Southeastern Michigan Raingage Network, of which about 25 stations lie in and around the Clinton River Watershed. Streamflow records available for the 30 U.S. Geological Survey gaging stations on the Clinton River and tributaries extend to a maximum period of 41 years for the Main Branch Clinton River gage at Moravian Drive in Mt. Clemens.

Parameter Development and HEC-1 Calibration

Basin modeling consists of describing the topologic structure of a basin (drainage basin bound-

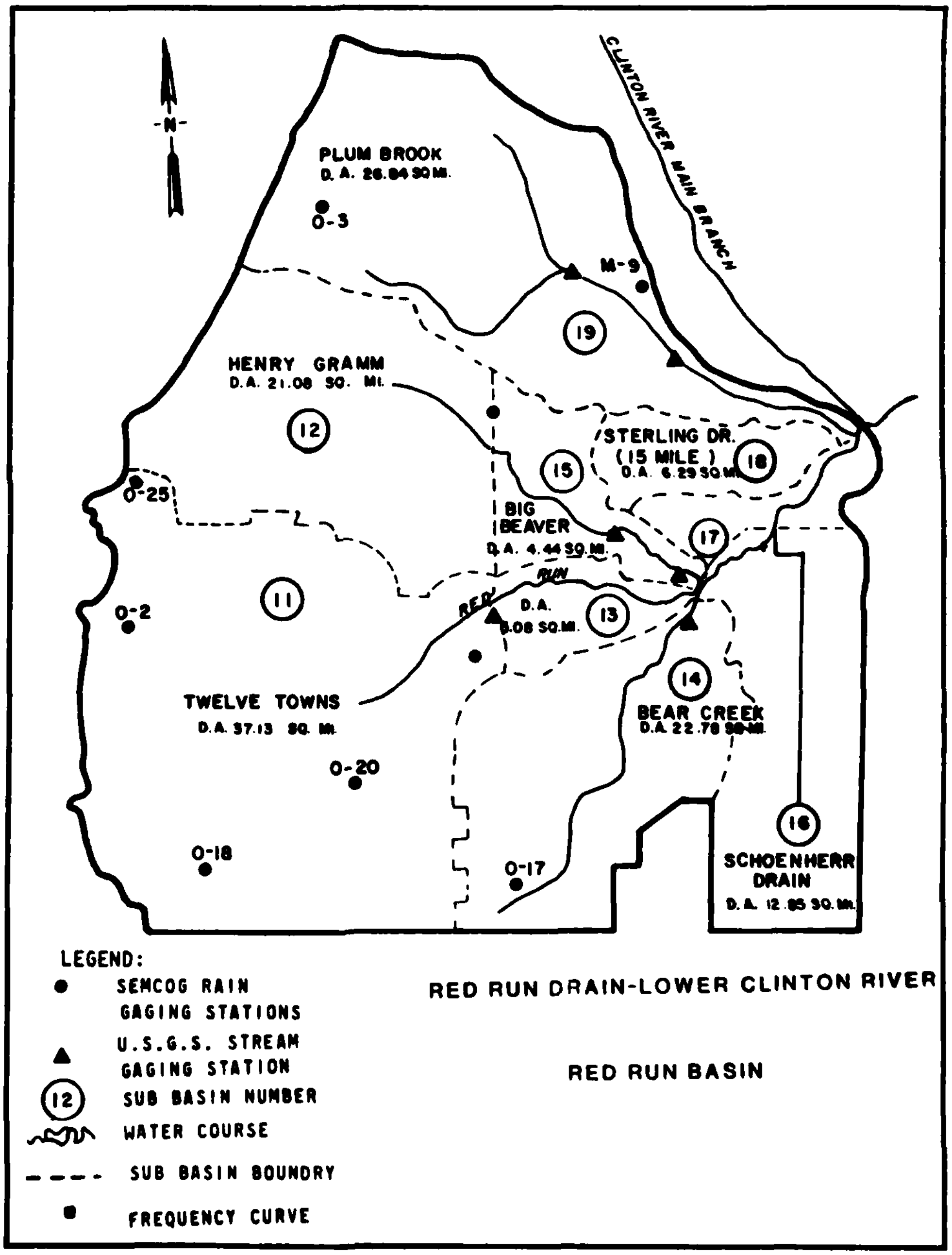

Figure 2. Red Run Drain

Table 1. Listing of U.S.G.S. Gages

GAGE NUMBER	STATION	YEARS OF RECORD
	Main Branch	
04161000	Clinton R. @ Auburn Hts.	19
04161100	Galloway Cr. nr. Auburn Hts.	16
04161540	Paint Cr. at Rochester	16
04168000	Stoney Cr. nr. Washington	17
	Red Run Watershed	
04162900	Big Beaver nr. Warren	17
04163400	Plum Brook at Utica	10
	Clinton River	
04164000	Clinton River nr. Fraser	28
04164500	N. Br. Clinton River nr. Mt. Clemens	28
04165500	Clinton River at Mt. Clemens	41

aries, stream channels, and the logical relationships between the drainage areas and stream channels) and defining the parameters that describe the precipitation-runoff response of the subbasins and the channels that comprise the basin. The basic driving input to the model is a time history of precipitation for each subbasin for the duration of the storm event under study. The basic output is computed flood hydrographs that represent a time history of the hydrologic response of the basins and stream channels under study.

Computer Program HEC-1 (8) uses a lumped-parameter approach for determining runoff from each subarea under consideration, i.e., the parameters and/or computations for a particular subbasin are assumed to apply for its entire area. Because of the many points of interest throughout the Red Run basin and the Lower Clinton River, the study area was divided into a total of 22 subbasins ranging in size from roughly 5 to 200 square miles as shown in the schematic diagram in Figure 3.

Unit Hydrograph Parameters - To develop the required unit hydrographs for past, existing and future land use conditions for the numerous subbasins of the Clinton River Watershed, regional relationships were *determined for unit hydrograph parameters*. In addition, unit hydrograph parameters for five rural subbasins were determined by optimization techniques from recorded data.

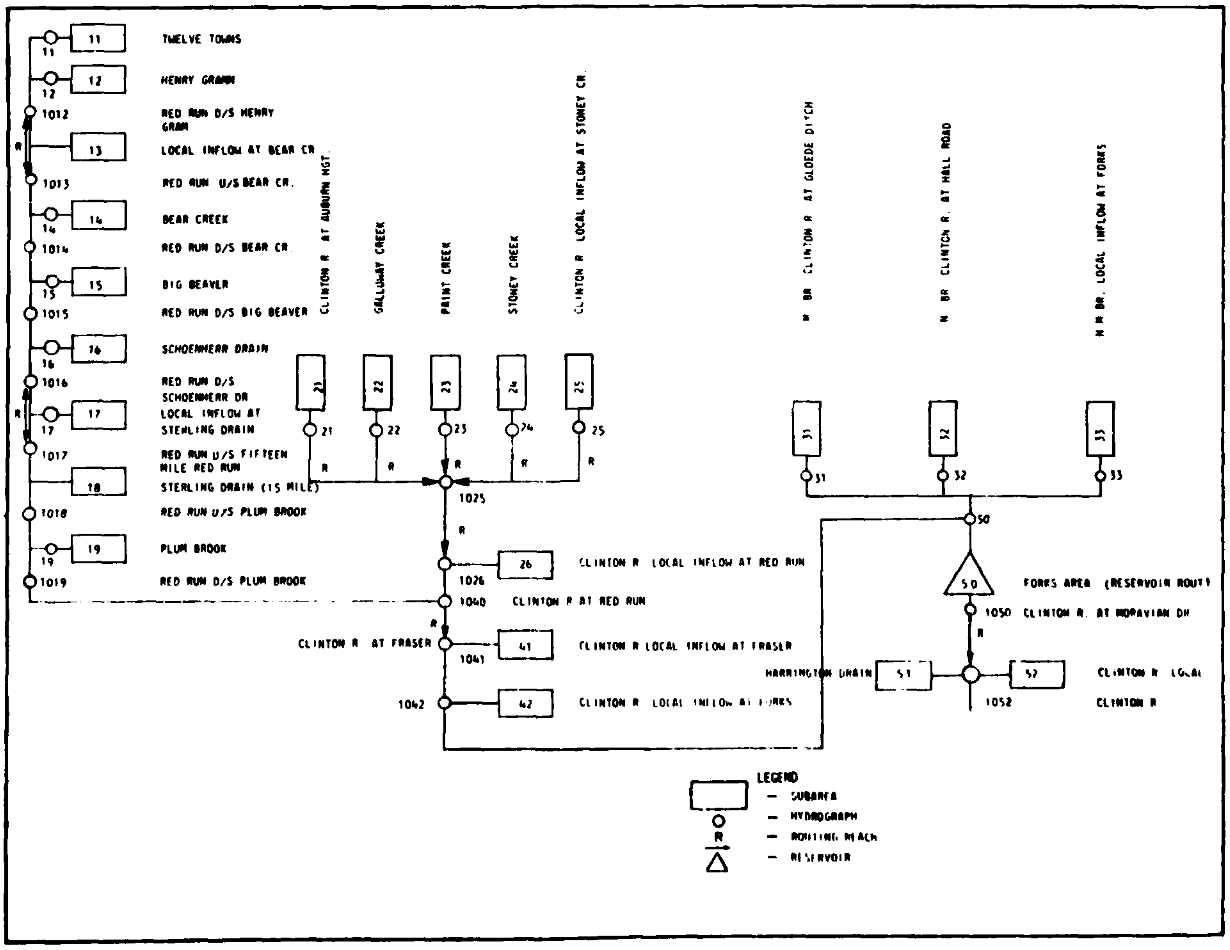

Figure 3. HEC-1 Schematic Diagram

Table 2. Drainage Basin Characteristics and Unit Hydrograph Parameters

Drainage Basin	USGS Gage No.	Area Sq. Mi.	Population Density Persons/ Sq. Mi.	Q_p (1) CFS	T_R (2) Hrs
Big Beaver Creek near Warren, MI	4-1629.0	23.5	774	799	13.0
Plum Brook near Utica, MI	4-1633.0	22.9	731	779	15.9
Rouge River Birmingham, MI	4-1660.0	36.9	1212	1107	12.5
Rouge River Southfield, MI	4-1661.0	88.0	1390	2288	15.0
Rouge River Detroit, MI	4-1665.0	185.0	2710	3700	27.0
Clinton River Mt. Clemens, MI	4-1655.0	734.0	1000	9836	38.0
McBride Drain near Macomb, MI	4-1644.5	5.8	114	223	9.3
Tupper Brook Ray Center, MI	4-1642.5	8.6	60	318	5.0
Evans Ditch Southfield, MI	4-1662.0	9.5	2376	603	12.0
East Branch Coon Cr. Armada, MI	4-1643.0	13.0	56	520	10.0
Gloede Ditch near Waldenburg, MI	4-1652.0	16.0	560	432	14.1
Upper Rouge River Farmington, MI	4-1663.0	17.5	1129	700	10.0
Red Run near Royal Oak, MI		36.5	7500	5110	4.0

(1) Q_p = unit hydrograph peak discharge.

(2) T_R = period of rise of unit hydrograph.

Criteria for relating unit hydrograph parameters to watershed characteristics for use in southeastern Michigan were given by Brater and Sherrill (2). Unit hydrograph parameters for 13 gaged basins in southeastern Michigan are summarized in Table 2. The unit hydrograph parameters in Table 2 are for average unit hydrographs developed from a number of storms for each basin. Extensive analysis by Brater and Sherrill showed that population density and drainage area were the most significant watershed characteristics in determining unit hydrograph parameters. Data from Table 2 were entered into HEC's multiple Linear Regression computer program to develop equations for unit hydrograph peak discharge, Q_p, and period of rise, T_r. The resulting equations are given below:

$$Q_p = \text{antilog } (1.85+.694\log A+.000099P_d) \quad (1)$$

$$T_r = \text{antilog } (.66+.359\log A-.000067P_d) \quad (2)$$

where:

Q_p=peak discharge of unit hydrograph in CFS

T_r=time of rise of unit hydrograph in hours

A=drainage area in square miles

P_d=population density in persons per sq. mile

The error in using equation 1 to estimate Q_p for the 13 basins ranges from .3% to 33%, with an average of 10%. The correlation coefficient for equation 1 is .991. The error in using equation 2 to estimate T_r for the 13 basins ranges from 4% to 95%, with an average of 24%. The correlation coefficient for equation 2 is .831.

Equations 1 and 2 were used to develop unit hydrograph parameters for the ungaged subbasins in the Red Run basin. Required population densities for the years 1940, 1950, 1960, and 1975 were obtained from census data. Population projections for the years 2000 and 2025 were obtained from the Southeastern Michigan Council of Governments. To facilitate use of HEC-1, values for Q_p and T_r from equations 1 and 2 were transformed to equivalent values of Snyder unit hydrograph parameters, t_p and C_p. Adopted values for these parameters are shown in Table 3.

Table 3. Unit Hydrograph Parameters and Impervious Areas for the Red Run Drain

Year		1940			1950			1960			1975			2000			2025		
Basin	Area Sq. Mi.	t_p (1)	C_p (2)	Imp. (3)	t_p	C_p	Imp.	t_p	C_p	Imp.	t_p	C_p	Imp.	t_p	C_p	Imp.	t_p	C_p	Imp
Twelve Town	37.13	10.2	.73	.04	7.5	.83	.07	5.1	.94	.10	4.8	.96	.10	4.7	.97	.10	4.6	.97	.11
Henry Graham	21.08	-	-	-	-	-	-	-	-	-	8.2	.70	.05	7.8	.71	.05	7.3	.71	.05
Local Inflow at Bear Creek	5.08	7.7	.45	0	7.5	.52	0	5.0	.61	.04	3.0	.70	.05	2.3	.75	.09	2.1	.76	.10
Bear Creek	22.78	13.7	.58	0	13.5	.58	0	9.1	.68	.04	5.8	.81	.08	5.1	.84	.08	4.0	.91	.10
Big Beaver	4.44	14.3	.58	0	14.1	.59	0	13.6	.59	.01	4.2	.62	.04	2.5	.71	.08	2.2	.73	.09
Schoenherr	12.85	10.9	.56	0	10.8	.56	0	10.4	.57	.01	4.7	.76	.09	4.1	.79	.08	3.5	.82	.09
Local Inflow at Sterling - Relief		5.8	.49	0	5.7	.50	0	5.5	.50	.01	3.6	.60	.05	2.7	.65	.07	2.2	.68	.08
Sterling Dr.	6.29	-	-	-	-	-	-	-	-	-	3.6	.60	.05	3.9	.68	.06	2.8	.75	.09
Plum Brook	26.84	16.0	.60	0	15.7	.60	0	15.1	.61	.01	12.7	.60	.03	9.5	.70	.04	9.1	.71	.04

(1) t_p = Snyder "Standard" Lag in Hours

(2) C_p = Snyder Peak Discharge Coefficient

(3) Imp. = Hydrologically significant impervious area expressed as a proportion of subbasin area

Infiltration Analysis - Hydrographs recorded for eight subbasins were analyzed to determine average soil infiltration capacity. In addition, data from other subbasins of the Clinton River and the adjacent River Rouge were analyzed and published by Brater and Sherrill (2). There is considerable scatter around the average seasonal values, depending primarily on antecedent rainfall and evapo-transpiration. The adopted average values for infiltration were .40 inch/hour for summer conditions and .10 inch/hour for winter conditions.

Impervious Areas. An important effect of urbanization is to create additional impervious area. An estimate for the area of impervious surfaces may be made from aerial photographs, but all precipitation on such impervious areas does not contribute to surface runoff. For this reason, Brater (2) employed the concept of a "Hydrologically Significant Impervious Area" (HSIA), which represents the percent of the total area which always contributes a runoff equal to the total precipitation minus retention, or, in other words, almost 100% runoff. Values for HSIA can be derived from the analysis of hydrographs resulting from large storms occurring after a long summer dry spell, when the infiltration capacity of the pervious portion is very high and therefore the entire surface runoff is derived from the Hydrologically Significant Impervious Area. In southeastern Michigan, the HSIA is approximately twenty five percent of the gross physical impervious area as determined from aerial photographs. Brater showed a correlation between HSIA and population density, P_d, for twelve watersheds in southeastern Michigan (2). This relationship is as follows:

$$HSIA = 1.38\, P_d \qquad (3)$$

where:

P_d = the population density in thousands of persons per square mile

HSIA is in percent of total area

Precipitation - In order to simulate the hydrologic response of the Clinton River Basin using the HEC-1 program, it was necessary to review and collect the pertinent precipitation data within the region for a number of historic storms. Precipitation amounts were input into the HEC-1 program on a subbasin basis using the Thiessen polygon or isohyetal method, depending on the extent of the data available.

Stream Flow Routing - Stream flow routing was done using the Modified Puls methods. With the Modified Puls method, discharge is considered a unique function of storage. A number of reaches were established for the Main Branch of the Clinton River and the Red Run Drain. Storage volumes were determined for each reach using the HEC-2 Water Surface Profiles Program. (11) Discharge-storage relationships were then developed to provide the required input for the HEC-1 program.

Calibration and Verification - Six separate HEC-1 data models were developed in this study, representing different stages of the development and urbanization of the Clinton River Watershed. These models represent conditions in the 1940's, 1950's, 1960's, 1970's, year 2000, and year 2025. The eight historic storm events listed in Table 4 were simulated to verify that the parameters used were reasonable. From gage records, flood hydrographs were determined for the Clinton River at Fraser, which includes runoff from both the Upper Clinton

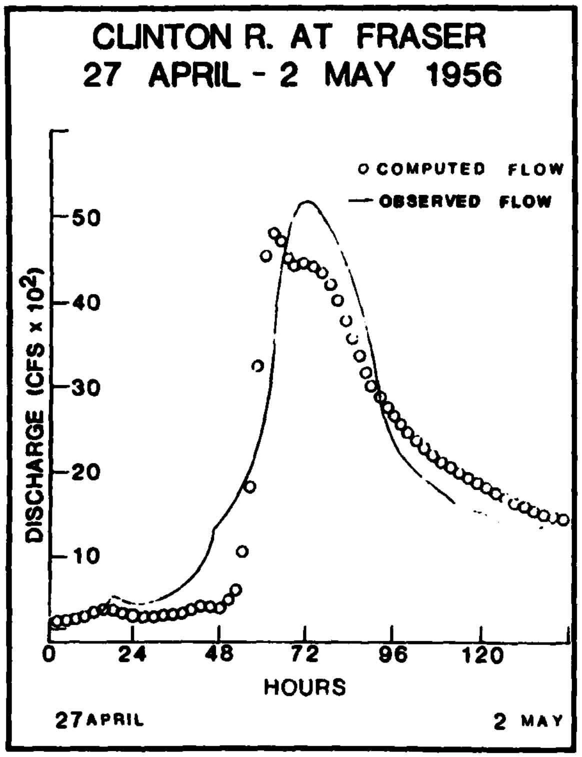

Figure 4. Hydrogr[illegible] Reproduction

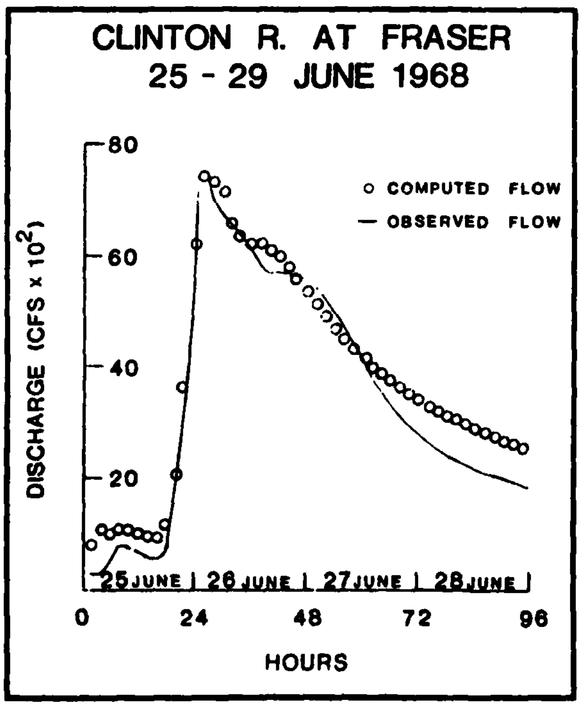

Figure 5. Hydrograph Reproduction

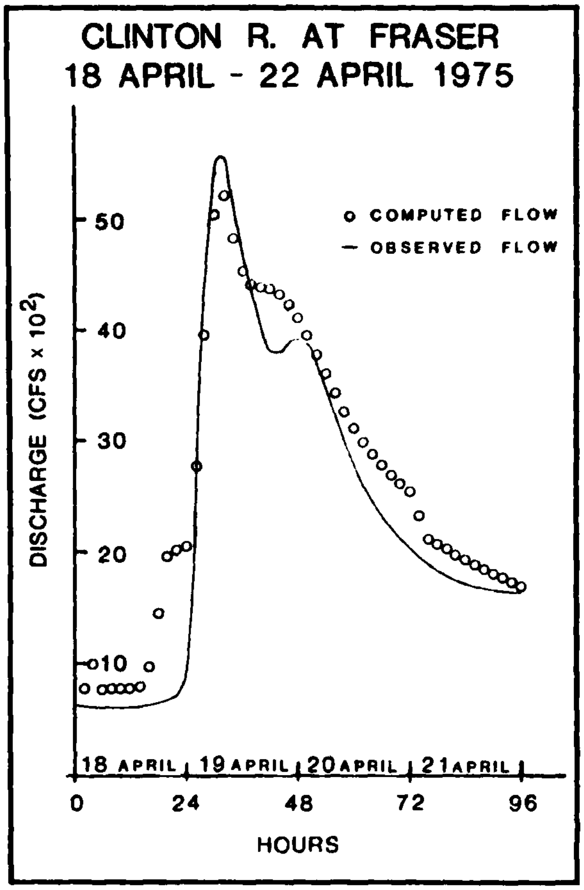

Figure 6. Hydrograph Reproduction

River and the Red Run Drain; the North Branch at Mt. Clemens, which includes runoff from the North Branch; and the Clinton River at Mt. Clemens, which includes runoff from almost the entire Clinton River Basin. The rainfall associated with each historic event was input to HEC-1 and computed flood hydrographs were compared to the observed hydrographs. The infiltration parameter DLTKR, which is an index of antecedent moisture conditions, (8) was calibrated within reasonable limits to provide the best hydrograph reproduction. Typical hydrograph reproductions are shown in Figures 4, 5, and 6.

Frequency Curves At Gaged Sites

Two key stream gage sites in this study are the Clinton River near Fraser and the Clinton River at Mt. Clemens. The Fraser gage has been in operation since 1947 and measures runoff from a 444 square mile area that includes the Upper Clinton River and the Red Run Drain. The Mt. Clemens gage has been in operation since 1934 and measures runoff from 734 square miles, which is nearly the entire Clinton River basin. Records from these two gages were of substantial importance in this study for calibrating the rainfall-runoff simulation model, and for assigning the exceedance frequencies to synthetic storm-loss rate combinations.

Table 4. Historic Storm Events Used for Verification and Calibration

Date of Storm	Observed Peak Discharge at Fraser (cfs)	Observed Peak Discharge at Mt. Clemens (cfs)
29 August - 2 September 1975	4,400	5,200
19-22 April 1975	5,500	12,100
4-9 March 1974	3,800	10,200
26-30 June 1973	4,400	3,900
25-29 June 1968	7,000	11,000
27 April - 2 May 1956	5,200	13,500
10-13 May 1948	8,000	14,500
3-8 April 1947	-	21,200

The time series of annual peak discharges at each site is nonstationary and nonhomogeneous - nonstationary because of the time-dependent effects of urban development; nonhomogeneous because in some years a winter event produces the annual maximum discharge, whereas in other years a summer event produces the annual maximum. Winter runoff events are characterized by cyclonic storms and very low loss rates; summer events by thunderstorms and relatively large loss rates. The following procedures were used for transforming the recorded annual peak discharges at Fraser and Mt. Clemens into consistent series that reflects existing land use conditions and channel characteristics (12).

As indicated previously, input data sets for HEC-1 were developed and calibrated to reflect existing watershed conditions and conditions in 1960, 1950, and 1940. Synthetic winter and summer storm events were prepared using storm patterns (i.e., time distributions) developed by Brater and Sherrill (2). They found that:

> "Typical large rainstorms in Southeastern Michigan have a nearly symmetrical time distribution with the maximum hour near the center of the rain storm. In winter the maximum hour contains about 24 percent of the rain and in summer the maximum hour usually provides about 55 percent of the total storm rainfall . . ."

The magnitude of a "base" storm was fixed by adopting a 25-year recurrence interval, 24-hour dur-

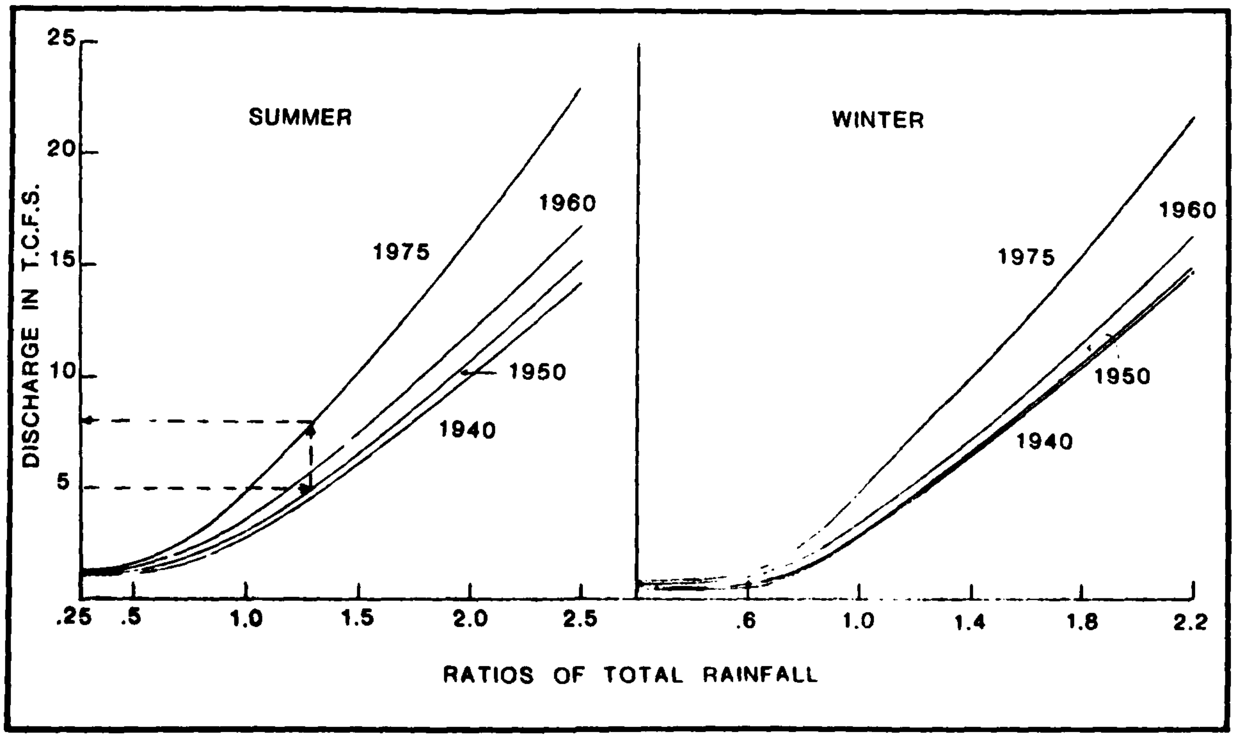

Figure 7. Peak Discharges vs Ratios of Total Rainfall at the [illegible] Gage

ation precipitation depth from NOAA's Technical Paper No. 40 (9). The recurrence interval of 25 years was arbitrary. It was not assumed in the approach described herein that the frequency of runoff is equal to the frequency associated with the synthetic storm. Rather, the purpose was to establish a base storm for use in subsequent steps in the analysis.

Six synthetic storms for each season, summer and winter, were developed by applying scaling ratios to the base storms. Ratios were selected so that peak discharges resulting from the storms would cover the range of discharges required to define discharge-frequency curves at the Fraser and Mt. Clemens gage locations. Winter synthetic storms were developed using scaling ratios of .2, .6, 1.0, 1.4, 1.8 and 2.2. Summer synthetic storms were developed with ratios of .25, .5, 1.0, 1.5, 2.0, and 2.5.

The multiplan-multiratio capability of HEC-1 permitted calculation of discharge hydrographs for six storms and four sets of runoff parameters (for watershed conditions in 1975, 1960, 1950, and 1940) in one run of the program. Hence two runs were required - one for summer conditions and one for winter conditions. Resulting relationships for calculated peak discharge versus scaling ratio for the Fraser and Mt. Clemens gages are shown in Figures 7 and 8. The relationships were used to transform each recorded annual peak discharge (for each season) to an existing-condition annual peak. This was achieved by entering the appropriate set of curves horizontally with the recorded peak discharge to the intersection with a curve (or interpolated curve) for the year in which the discharge was recorded, and then moving vertically to an intersection with the 1975 curve, from which the transformed peak discharge could be determined. The largest of the two transformed peaks for each year was incorporated in a time series of annual peaks. Statistical analyses of the resulting series was performed using techniques in Bulletin 17 of the Water Resources Council (10). The resulting frequency curves are shown in Figures 9 and 10. Also shown are frequency curves based on observed (untransformed) peak discharges. Urbanization has had a substantial influence on the frequency curve for Fraser, whereas the influence at Mt. Clemens is minor.

Seasonal Frequency Curves At Fraser

Subsequent development of discharge frequency curves for ungaged sites required separate frequency curves for winter and summer conditions for the Clinton River at Fraser. The curves are shown in Figure 11. The annual curve is the same as shown in Figure 9. The winter curve was obtained graphically using Weibull plotting positions for the transformed winter peaks. The series of summer peaks was truncated because for a number of years the observed summer peaks were smaller than the base value used by the U.S. Geological Survey as a basis for publishing instantaneous peaks. The summer frequency curve was therefore obtained by subtraction using the following equation:

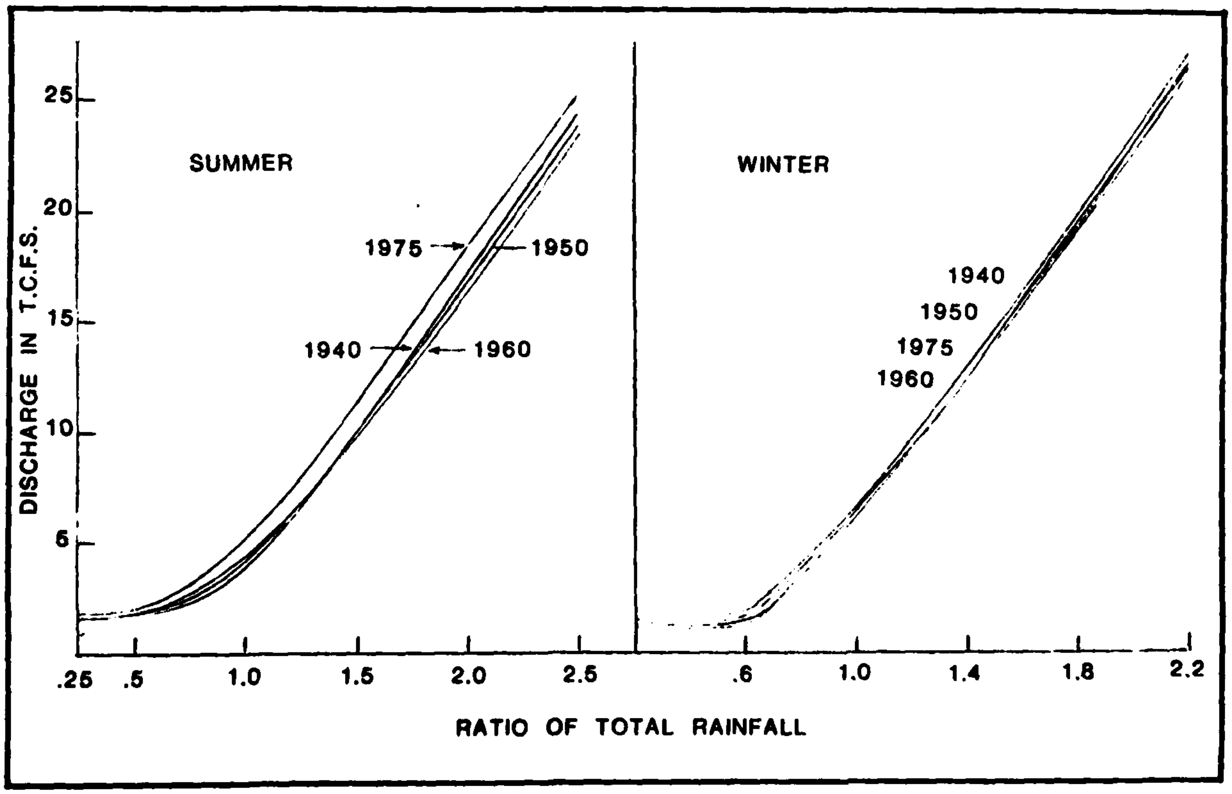

Figure 8. Peak Discharges vs Ratios of Total Rainfall at the Mt. Clemens Gage

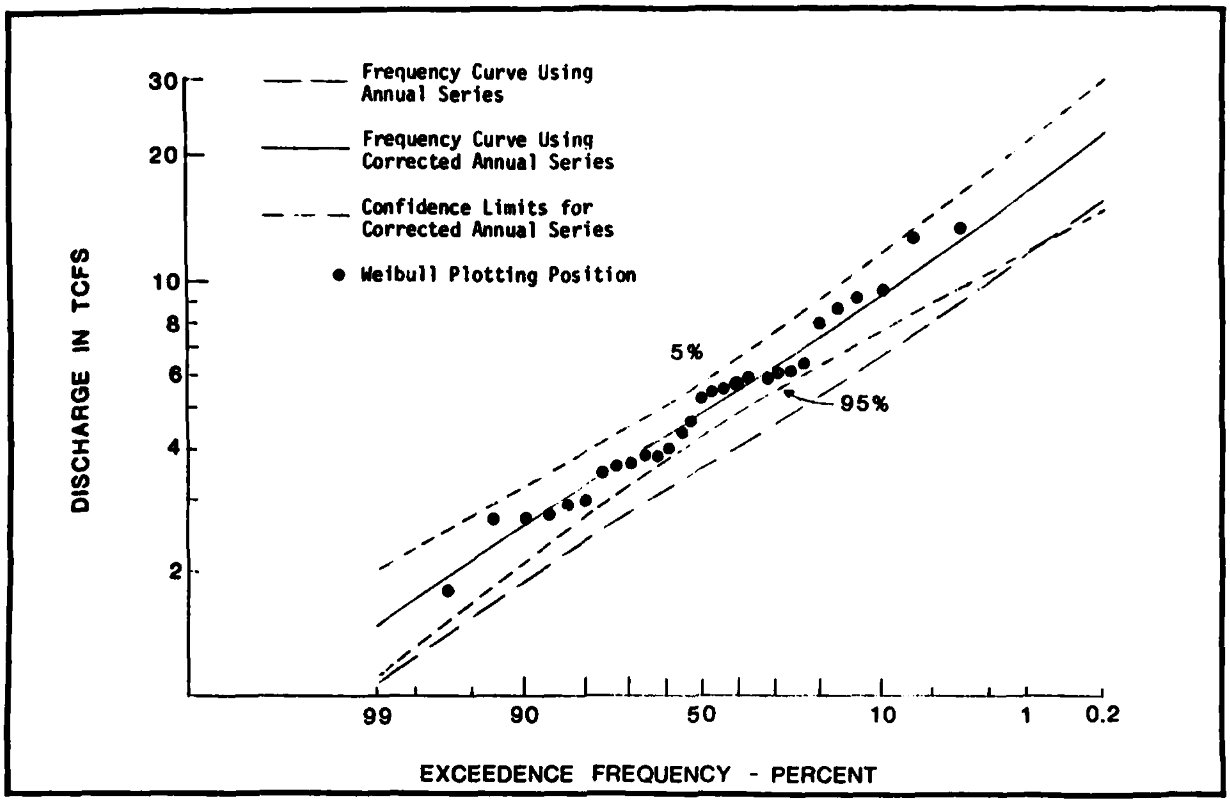

Figure 9. Frequency Curve at Fraser

$$P_{SUMMER} = \frac{P_{ANNUAL} - P_{WINTER}}{1 - P_{WINTER}} \qquad (4)$$

where:

P_{SUMMER} = Exceedance probability for a given discharge on the summer-season frequency curve.

P_{WINTER} = Exceedance probability for a given discharge on the winter-season frequency curve.

P_{ANNUAL} = Exceedance probability for a given discharge on the annual (all-season) frequency curve.

The available transformed summer peaks are plotted in Figure 11 for comparison using Weibull plotting positions.

Frequency Curves At Ungaged Sites

The basis for developing peak-discharge frequency curves at ungaged sites throughout the 140 square mile Red Run Basin was to use balanced synthetic storms as inputs to calibrated HEC-1 models for the basin. Synthetic storm-loss rate combinations were assigned exceedance frequencies in accordance with exceedance frequencies of calculated peak discharges obtained from the previously developed frequency curves for Fraser.

Synthetic winter and summer storms were based on criteria from NOAA's Technical Paper No. 40 (9) and the time distributions developed by Brater and Sherrill (2). Application of areal adjustments from Technical Paper No. 40 to point rainfalls of various durations results in storm hyetographs that are a function of drainage area size. Therefore, separate HEC-1 runs were made using storm hyetographs associated with drainage areas of 10 square miles and 400 square miles. Peak discharge for any particular (ungaged) drainage area size was obtained by logarithmic interpolation between the peak discharge associated with the storm hyetographs for 10 square miles and 400 square miles.

Winter and summer frequency curves were developed at all ungaged locations of interest, for existing land use conditions and for land use conditions in the years 2000 and 2025, using the techniques described above. Winter and summer frequency curves for a given land use condition were then combined to obtain an annual curve based on the probability of union of independent events. Figure 12 shows a typical set of frequency curves for an ungaged location.

Findings

The Red Run Drain and Lower Clinton River watersheds have undergone urbanization since the 1940's. The effect of this urbanization on the peak discharges recorded at the Fraser and Mt. Clemens gages can be seen in Figures 7 and 8. Urbanization has had a substantial influence on the frequency of discharge

for Fraser, as shown in Figure 9. The peak discharge at Fraser is comprised almost entirely of runoff from the Red Run Drain, which peaks much sooner than runoff from the main branch of the Clinton River. The Red Run has been substantially urbanized, and so this change in frequency is not unexpected. The influence of urbanization at Mt. Clemens is minor as shown in Figures 8 and 10. Peak discharges at Mt. Clemens are a combination of runoff from the North Branch Watershed, largely rural, and the Red Run Drain. Urbanization has decreased the peaking time of the Red Run Drain, which tends to move apart the time of occurrence of the peak runoff rates of the North Branch and the Red Run Drain. This phenomenon has tended to counteract the increase in peak discharge rates due to urbanization.

Summary

The development of peak-discharge frequency estimates at ungaged locations in an urbanizing watershed for existing and future conditions is, in our opinion, one of the most challenging tasks a hydrologist can face. Three factors aided substantially in the conduct of the study; (1) availability of results of previous extensive studies pertaining to the hydrology of Southeastern Michigan, (2) a relative abundance of precipitation data and (3) the finding of a statistically significant index of urbanization, population density, to which runoff parameters could be related in a convenient fashion.

The basic approach was to use a single event rainfall-runoff model to transform nonstationary, nonhomogeneous series of recorded annual peak discharges to consistent series representing existing and future conditions. The problem of nonhomogeneity was handled by using separate analyses for the summer and winter seasons. Nonstationarity was treated by developing a set of storm magnitude vs. peak discharge relationships for various points in time and using these relationships to transform nonstationary series to stationary ones. In utilizing synthetic storms to develop peak discharges for ungaged locations, storm-loss rate combinations were assigned exceedance frequencies equal to the exceedance frequencies of calculated peak discharges at the gage at Fraser.

Acknowledgements

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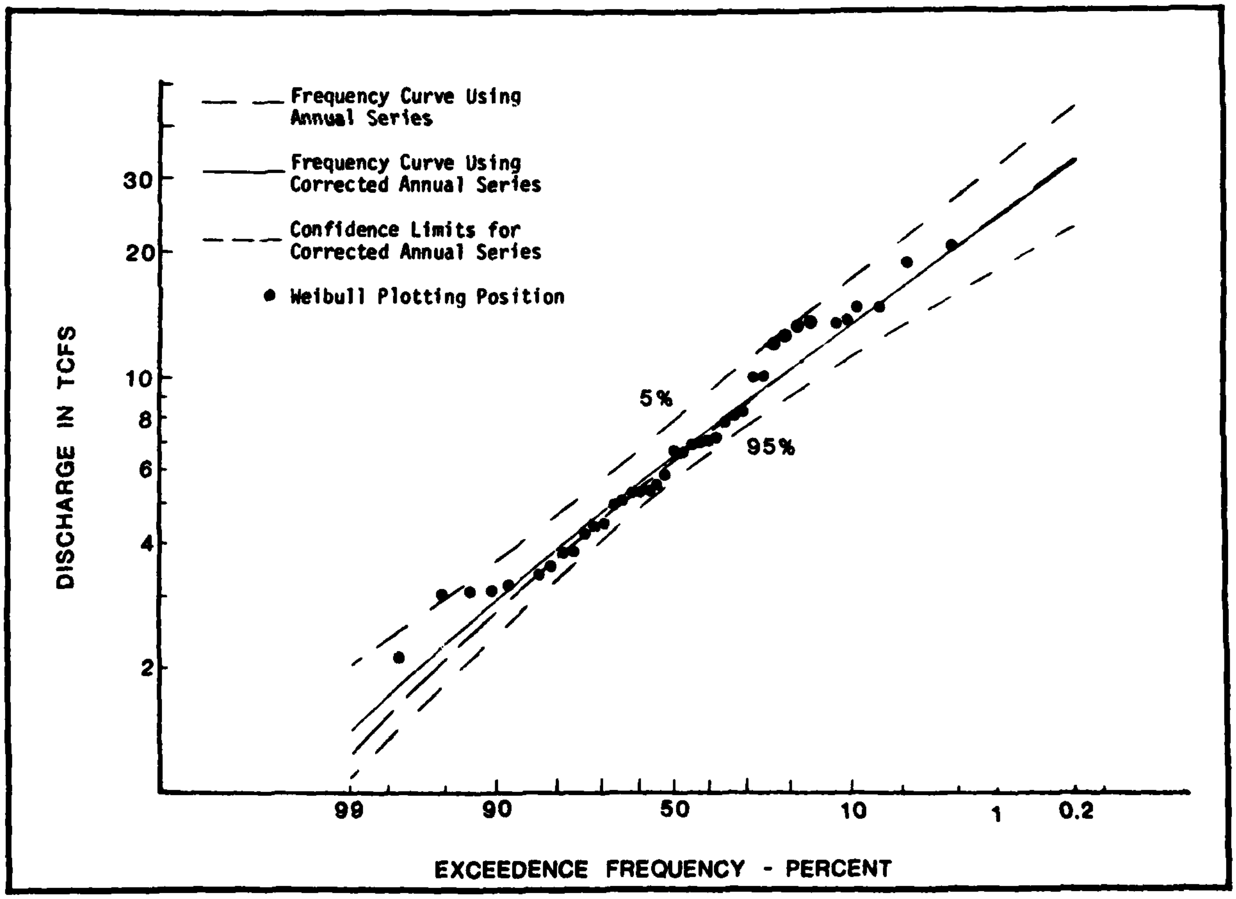

Figure 10. Frequency Curve at Mt. Clemens

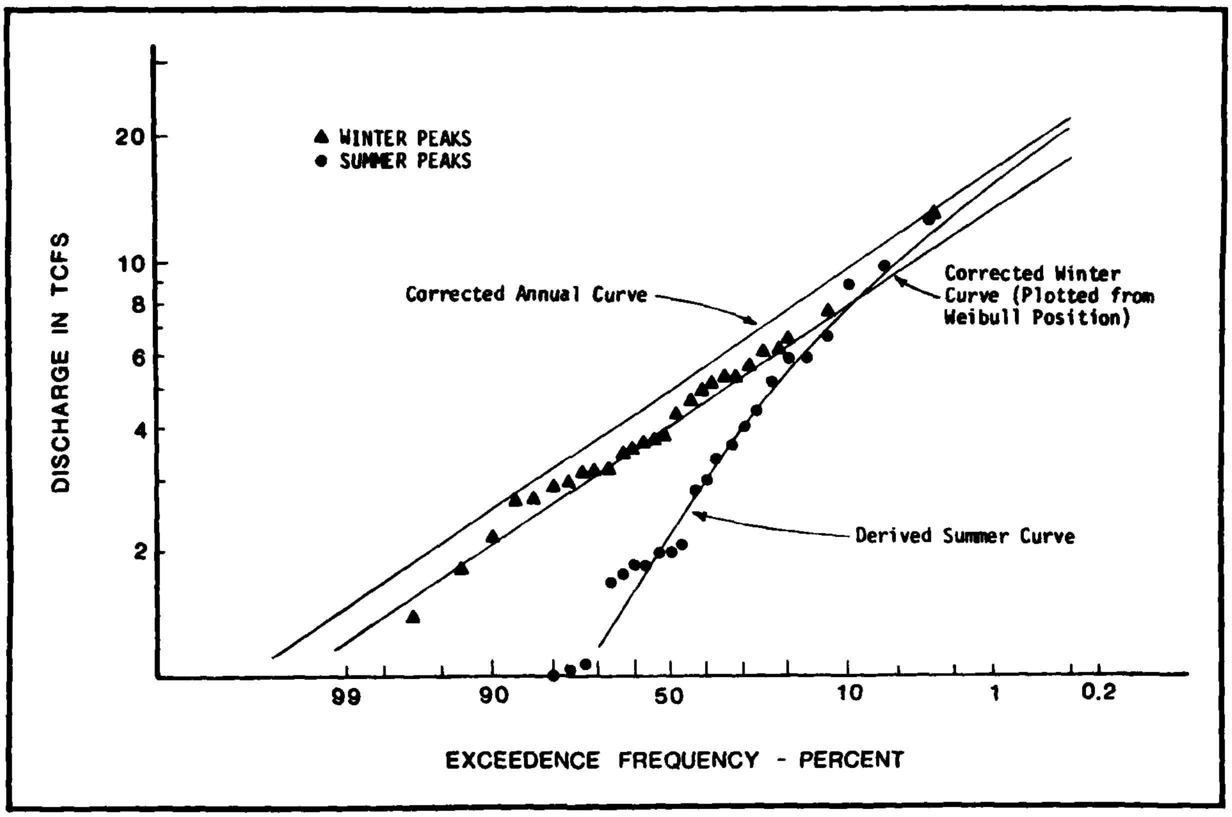

Figure 11. Seasonal Frequency Curves at Fraser

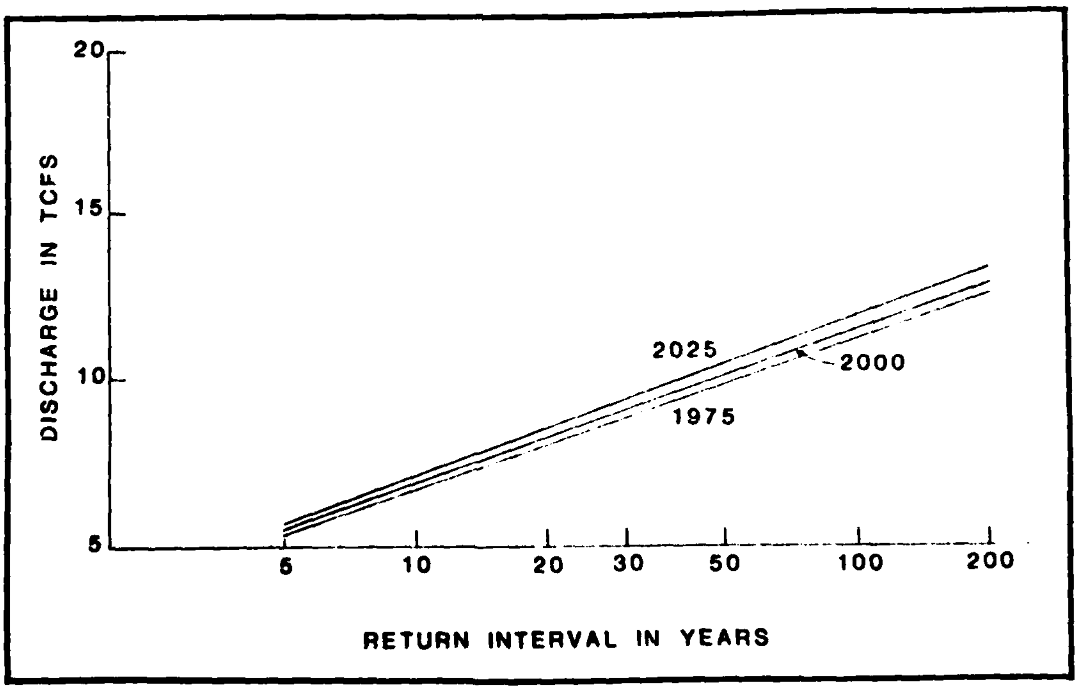

Figure 12. Typical Frequency Curves at an Ungaged Location

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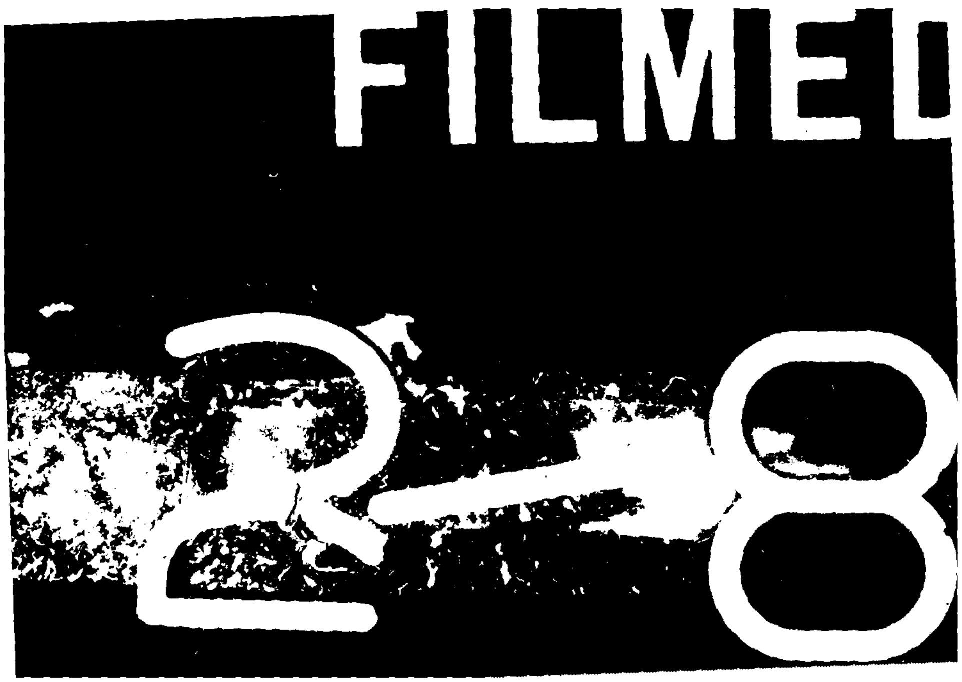
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